Chenani-Nashri Tunnel, the longest road tunnel in India: a challenging case for design-optimization during construction

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ABSTRACT: The 9km-long Chenani-Nashri Tunnel, currently under construction, is the longest road tunnel in India and it is part of the planned four-lane widening of the NH-1A between Udhampur and Banihal, in the state of J&K. Bypassing the existing NH-1A from km 89 to km 130, the tunnel crosses a sub-Himalayan formation with a maximum overburden of 1050 m. With an escape tunnel running parallel to the main tunnel, excavation is performed with D&B method by Leighton-Welspun Contractors. Geodata Engineering (GDE) provides consultancy services for detailed design and construction supervision, including 3D geotechnical monitoring. Back-analyses of already-excavated sections were performed to understand the causes of unexpected responses of the flysch rock mass. The numerical models were fed with actual geological-geomechanical conditions encountered during excavation and monitoring results, in order to evaluate the combined effects of the field stress with the geostructural conditions of the rock mass. The 3D Monitoring System, specifically implemented by GDE for this project, has played a key role in understanding the real rock mass behaviour, allowing to highlight potential risks, to select the proper tunnel support class, to check the effectiveness of the countermeasures and provide cost-effective solutions to speed-up the construction process.

1   Project description

The Indian Government has entrusted the National Highways Authority of India (NHAI) with the responsibility for rehabilitating, strengthening and four-laning of Chenani to Nashri Section of NH-1A, from km 89 to km 130, namely "Highway Project" (Figure 1) which is the most important and recent Indian project planned to connect the Kashmir valley with the rest of the Indian transportation network.
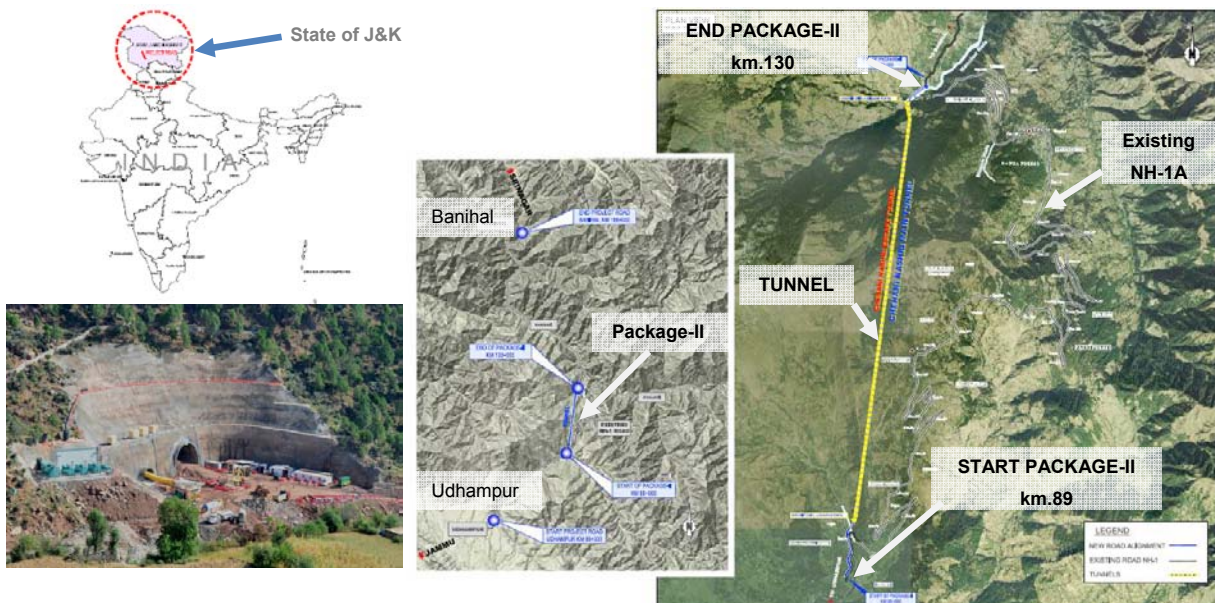

Figure 1. Site of the "Highway Project"

In addition to the underground works, the project includes surface works comprising 1.3 km approach road to the South portal and about 0.6 km of road approaching the North portal, two bridges, toll plazas, all project facilities and spoil dumps.

The Chenani-Nashri system includes a 9 km long bi-directional (2-lane) main tunnel with a separate parallel escape tunnel designed as emergency access for the rescue teams during operation as well as for site traffic use during construction stage. These two tunnels are connected through pedestrian cross passages every 300 m and vehicular cross passages at intervals of 1200 m.

For the main tunnel (Figure 2) the total width of the clearance profile is 11.75 m, including the 9.35 m wide paved carriageway and 1.2 m wide walkways on both sides. The tunnel geometry has been defined taking also into account the provisions for the fully transverse ventilation system (exhaust air and fresh air ducts are provided). The escape tunnel geometry has been defined in conjunction with the contractor on the basis of a detailed study on the available construction equipment, such as jumbos, excavators and dumpers. The escape tunnel complies with the required clearance dimensions for the operational stage (5.0 m width) with fully "motorable" footpaths on both sides. Cross passages allow the transit of machines and equipment, therefore it is possible (if necessary, depending on the construction planning) to use them to open additional excavation faces for the construction of the main tunnel, which allows to take advantage of the faster excavation rate of the escape tunnel compared to the main tunnel.
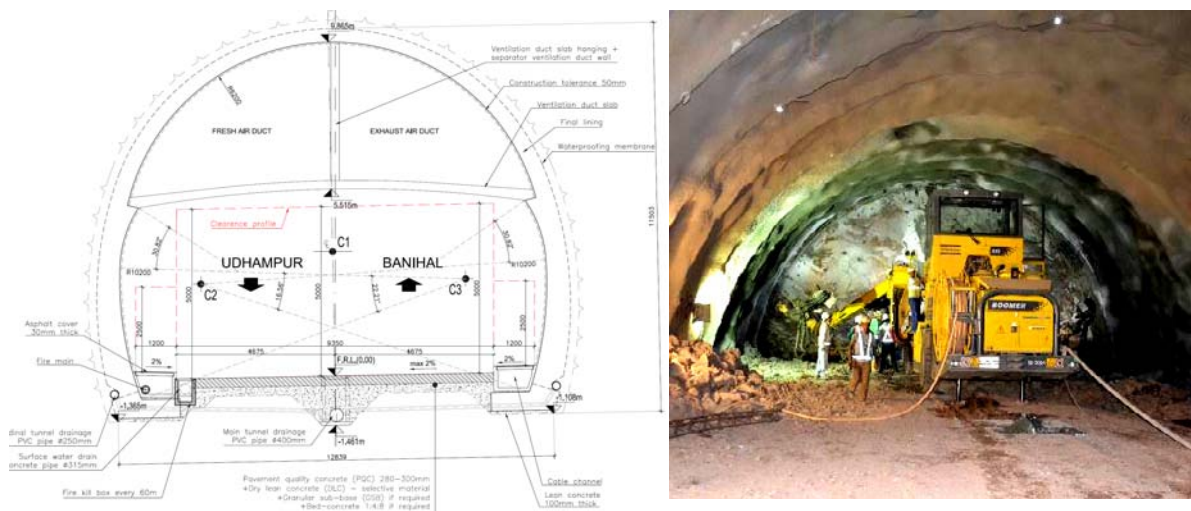

Figure 2. Main Tunnel typical section and jumbo used for the top heading excavation

The Chenani-Nashri tunnel is excavated by conventional method, namely Drill and Blast. The excavation face is sub-divided into top-heading, bench (right half and left half, alternatively) and invert (if necessary). Mechanical excavator is likely to be used in the sectors, such as shear zones, where weak rock mass conditions are expected.

The excavation cross-section of the main tunnel varies from 130 m^2 to 170 m^2, depending on the support class. Primary lining includes shotcrete, rock bolts and lattice girders (where required) to immediately stabilize the rock mass after excavation. Final lining (varying in thickness and reinforcement as required) has been designed to withstand the predicted long-term loads and seismic loads, which are not negligible in proximity of the portals. Between primary and final lining a waterproofing membrane paired with geotextile protective felt is installed from the top of the tunnel up to the concrete foundation beams.

2   Geological context, rock mass classification and design criteria

The project area lies in the western Himalayan region, in the collisional belt sector which is better known as the sub-Himalayas. This tectonic domain is delimited in the South by the Himalayan main frontal thrust (MFT) and in the North by the main boundary thrust (MBT). The rock masses along the tunnel alignment belong to the Lower Murree Formation which includes a sequence of interbedded sandstone and siltstone/claystone layers (flysch) with thickness ranging from a few layers to 10m.

Given the high overburden, one of the main critical aspects during design stage was to define the relative predominance among the different lithologies (sandstone, siltstone and claystone) at a tunnel

scale. More generally, the design was mainly developed by the analytical probabilistic approach and the expected excavation behaviour was calculated taking into account variability and uncertainty of rock mass properties and *in situ* stress. Such a quantitative method allowed to assign the adequate support type to each geomechanical hazard.

The same criterion was consistently followed during construction stage, for the selection of the support to be installed at the tunnel face. Notably, the "GDE multiple graph" (Russo, 2008) has been systematically applied on site. An example, related to the case analyzed in the next section is shown in Figure 3. For this specific case a fictitious overburden was considered to simulate the anisotropic state of stress resulting by the back-analysis. It is important to observe that the field of application of each support class is reported in the graph, as resulting from the associated design combination of geomechanical properties. Therefore, by assessing the key classification parameters (i.e. GSI, intact rock strength σ_{ci}, in situ stress and RMR) the correct support section type is selected directly at the tunnel face. Moreover, it should be noted that in place of the quantitative assessment of the GSI in the first quadrant of the graph (at the bottom right side), the specific Hoek and Marinos chart for heterogeneous rock mass (2001, 2007, 2011) has been applied and the resulting GSI was directly entered in the second quadrant.
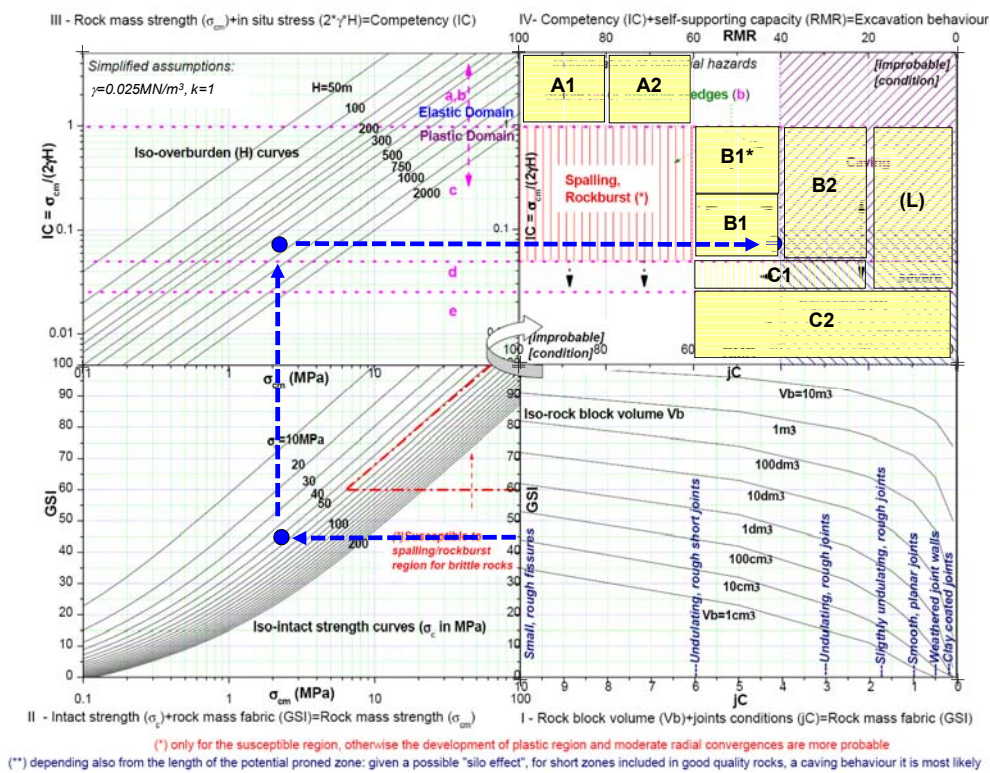

Figure 3. Application of the "GDE multiple graph" (Russo, 2008)

3   Analysis of an instability phenomenon during tunnel construction

A local gravitational instability involving the right crown/sidewall of the tunnel occurred on May 23th 2012, at approximately 820 m from the South portal (TM820). Some cracks in the shotcrete lining appeared immediately after the primary support installation, showing an unexpected response of the rock mass. Cracks were mainly localized on the right crown/sidewall of the tunnel where a claystone/siltstone layer would be constantly intercepted during top heading excavation, due to the unfavorable orientation of the flysch strata (almost sub-parallel to the tunnel axis).

According to the face mapping, the presence of weak interbedded siltstone and claystone layers was affecting the upper right side of the tunnel face. Three sets of prominent joints and few random joints were detected, which were closely to widely spaced with tight to open aperture. Some of the joints had silty-clay infilling.

In that sector the geomechanical classification in the top-heading excavation was initially indicating the requirement of the support class B1*, with 2.5-3.5 m pull length, 150mm fibre reinforced shotcrete

(FRS) and systematic bolting. Nevertheless, as described in the following, on the basis of the back-analysis, an adjustment of some classification parameters resulted necessary.
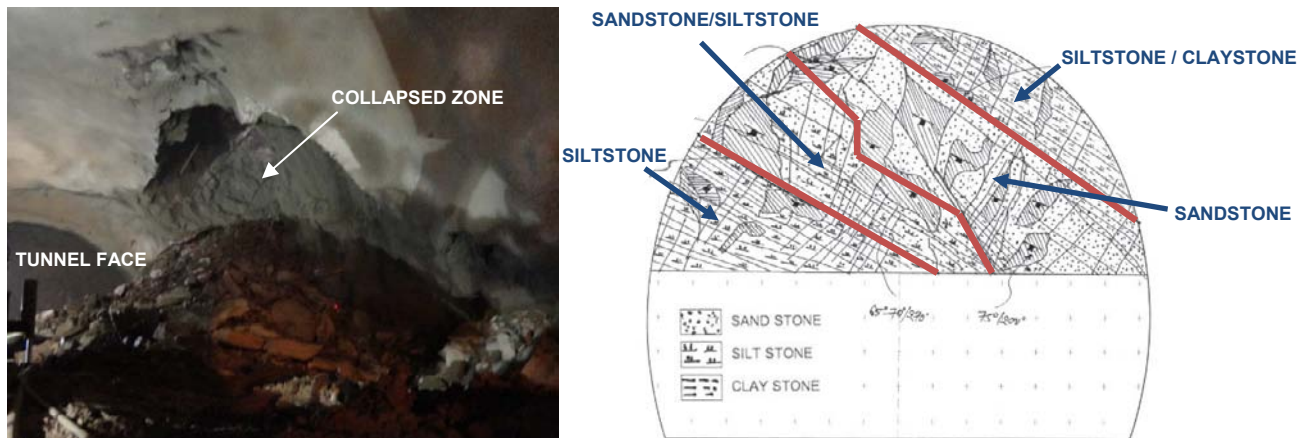

Figure 4. Collapsed area and geological face mapping at TM820

In order to better understand the causes which led to the cracks and to the instability, several back-analysis were performed using the recorded geotechnical and geostructural conditions and the available monitoring results. The Phase^2 numerical model hereinafter presented has been defined taking into account the strata orientation shown in the face mapping and the state of stress dependent on the overburden of 350 m. It should be noted that, in order to achieve results that were consistent with the monitoring data it was necessary to apply an anisotropic state of stress, obtained by rotating the stress-block and by increasing the main stress towards NE-SW by 50%. This anisotropy is reasonably due to the geomorphology of the ground surface, which inevitably affects the orientation of the field stress (Figure 5) in the South end. The key geomechanical parameters for each lithology were calculated through an iterative process and the figures reported in Table 1 were finally selected.
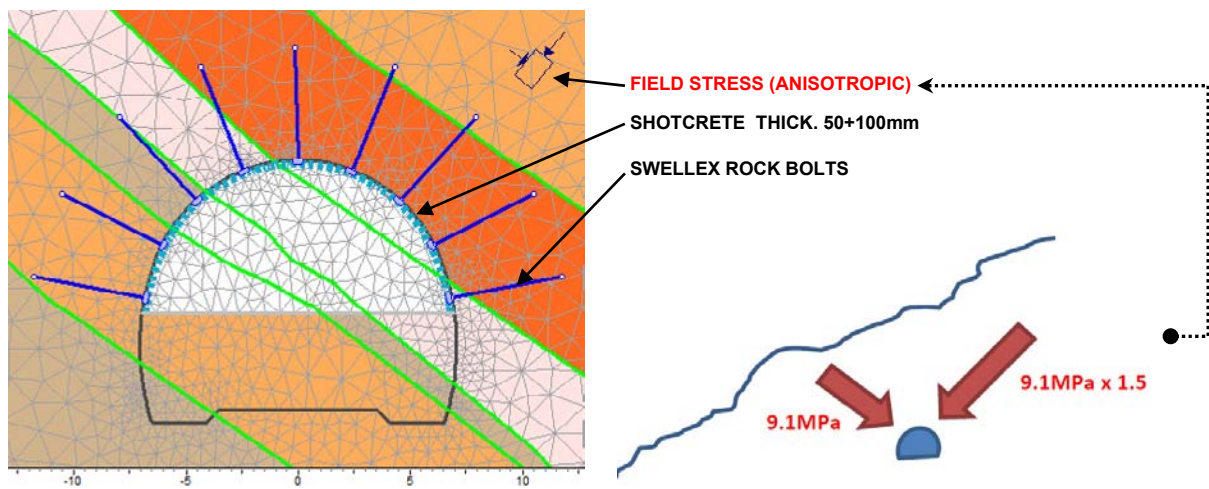

Figure 5. Numerical model in Phase^2 and justification of the anisotropic state of stress

Table 1. Geomechanical parameters assumed for the numerical analysis

Lithology	GSI [-]	UCS [MPa]	m_i [-]	m_b [-]	s [-]	a [-]	E [MPa]	ν [-]
Sandstone	45	100	11	1.5428	0.0022	0.5081	6150	0.25
Sandstone/Siltstone	45	75	10	1.4026	0.0022	0.5081	5032	0.25
Siltstone	45	50	9	1.2623	0.0022	0.5081	4193	0.25
Siltstone/Claystone	45	15	6	0.8415	0.0022	0.5081	839	0.30

The obtained results in terms of displacements, related to the stage in which the primary lining is completed, were initially compared with the available monitoring data (notably the chord length reduction). The estimated unrecorded displacements (occurring between excavation and targets

installation) were subtracted to the total displacements. The comparison (Figure 6) proves the reliability of the numerical model and allows to highlight the critical zone where cracks of shotcrete were initially recorded. The analysis moved forward with the comparison of the last available monitoring data recorded on May 23th 2012 (a few hours before the instability phenomenon) with the maximum calculated radial displacement. In this respect, a potential creeping behavior of the claystone/siltstone layer, reasonably accentuated by the presence of water, was assumed. The outcome proved very interesting, and the exact zone where the collapsed had occurred was identified by the analysis (Figure 6), thus offering a reasonable explanation of the interaction between the real state of stress and the observed rock mass geomechanical and geostructural conditions.
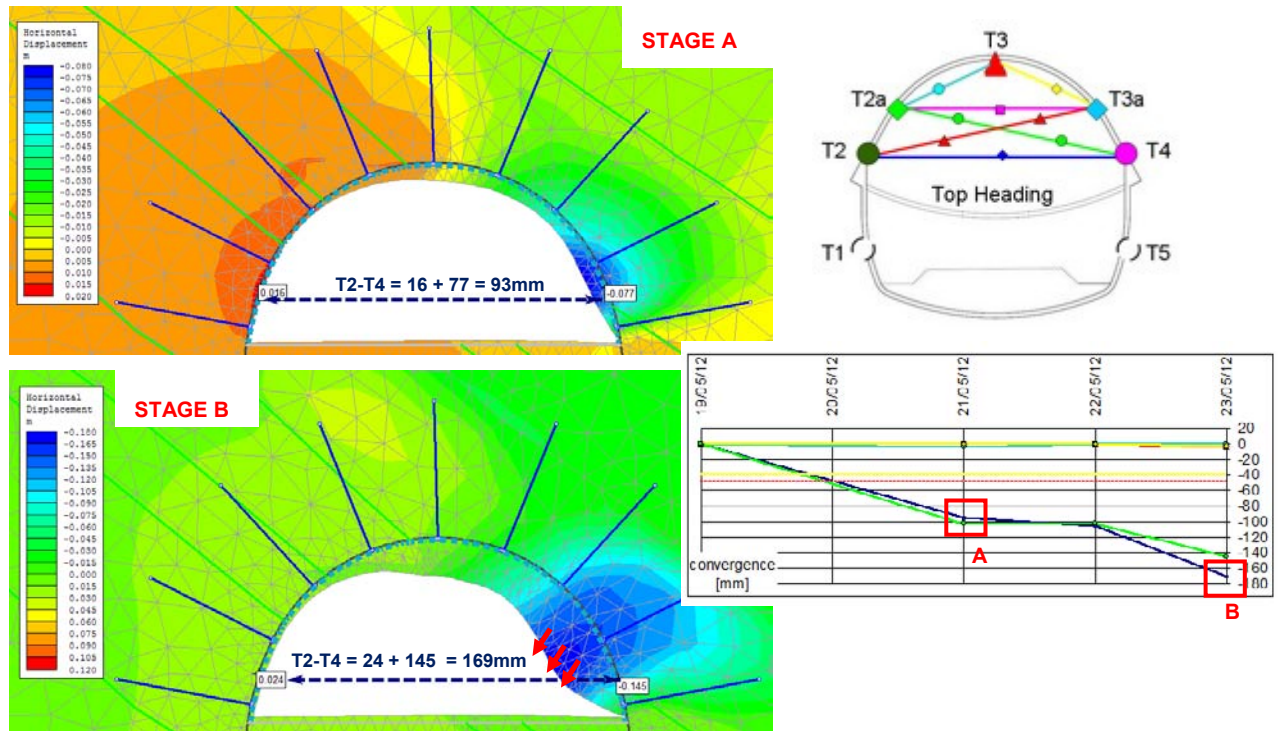

Figure 6. Comparison between the monitoring readings and the numerical analysis results
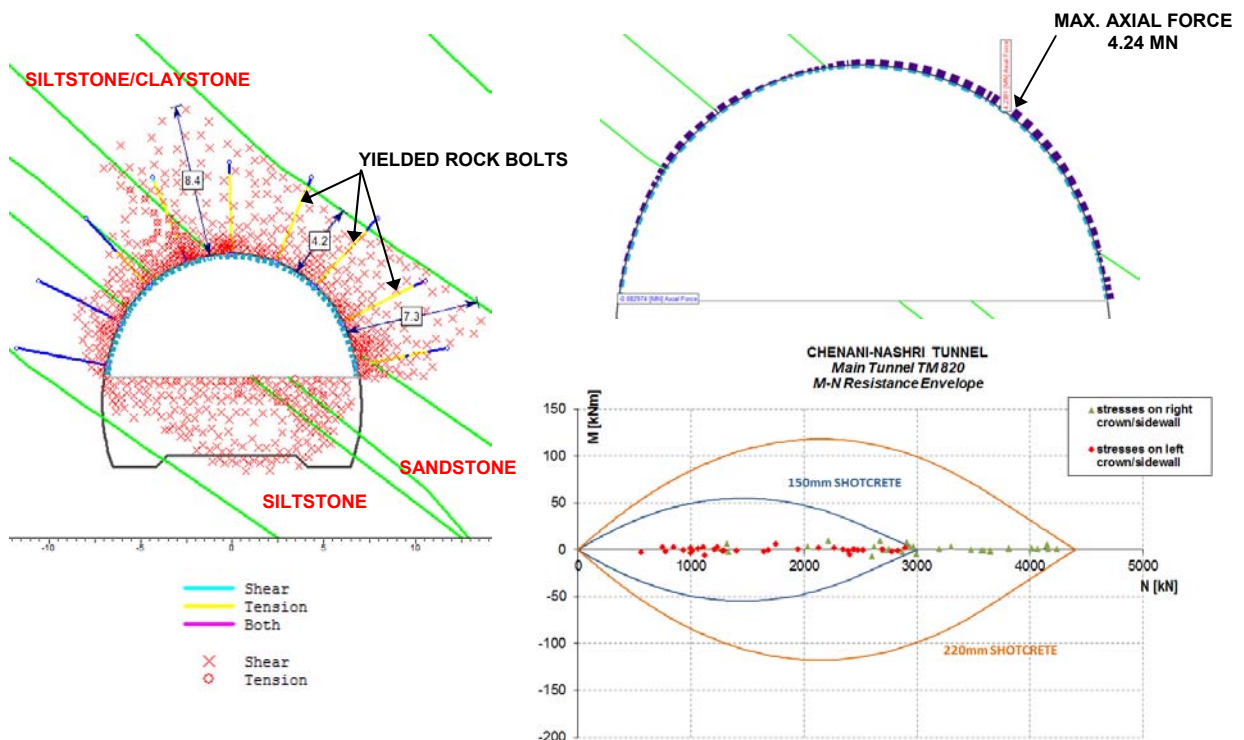

Figure 7. Yielded elements and N-M primary lining structural verification

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The yielded elements (Figure 7) are mainly concentrated in the weak siltstone/claystone layer on the right crown-sidewall as well as the Swellex rock bolts installed at the right sidewall are almost yielded. Rock bolts were still in place after the collapse, due to the residual capacity at the end of the rock bolts, which was confirmed by the numerical analysis. Highest stresses in the shotcrete were recorded on the right crown/sidewall, where the maximum compressive axial force in the lining is not compatible with the compressive strength of the 150mm shotcrete, causing the development of cracks as seen on site. Consequently, a stronger support was required as already confirmed by the application of the "GDE Multiple Graph" (B1 section with lattice girders and greater thickness of shotcrete, see Figure 3).

An additional optimization of the rock bolts pattern was also provided taking into account the orientation of the strata.

4   The importance of the Monitoring System during construction

GDE has implemented, specifically for the Chenani-Nashri Tunnel, a sophisticated as well as "easy to read" 3D-monitoring system (GDMS) which is very useful for understanding the response of the excavation and check the adequacy of the installed support.

A dual-level action plan was proposed, comprising a set of actions to perform, should the trigger limit (in terms of convergences) be exceeded:

- Attention level: a percentage of the predicted deformation. On exceeding this limit the frequency of readings will be increased in order to evaluate the deformation speed and the convergence trends. This low limit allows enough time to assess convergence trends and to decide appropriate countermeasures, should the movement continue to the alarm limit.

- Alarm level: the 100% predicted deformation, as calculated at design stage (coincident with the ultimate support capacity of the section). Surpassing this limit triggers the start of the procedure for action and countermeasure.

Convergence limits have been evaluated with a conservative approach. Only the ground reaction curves related to the probability of failure of the primary support equal to 0% ([C]-[D]>0) were taken into account. Such curves were estimated through a capacity-demand analysis, to check the reliability of the primary lining for each support section type (Figure 8).
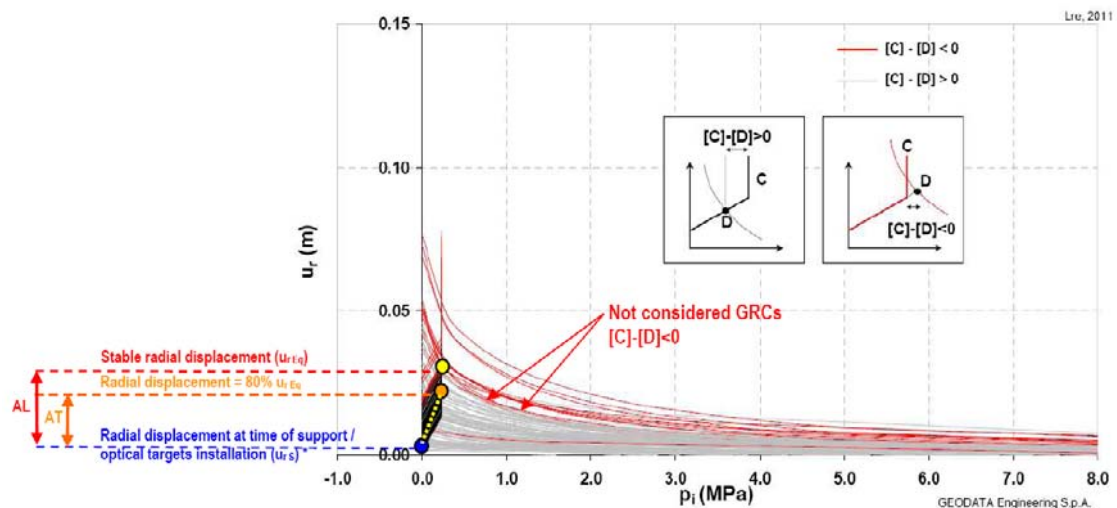

Figure 8. Criteria for the evaluation of the attention and alarm limit

Attention and alarm limits have been defined deducting from the maximum radial displacement ($u_{r,Eq}$), the amount of displacement ($u_{r,S}$) that occurs before the installation of the primary support. The assumption is that the optical target is installed at the same time of the primary support, so that the first part of the displacement cannot be recorded by the monitoring instruments.

The example presented in Figure 9 refers to the monitoring section at TM365 (class B1*, with 150mm FR shotcrete and rock bolts), which showed a 20mm chord displacement during top heading excavation (with the highest displacement at T4 target displacement), exactly in correspondence of the weak interbedded siltstone and claystone layers. Displacements increased by over 3 times during bench excavation, in the same location.
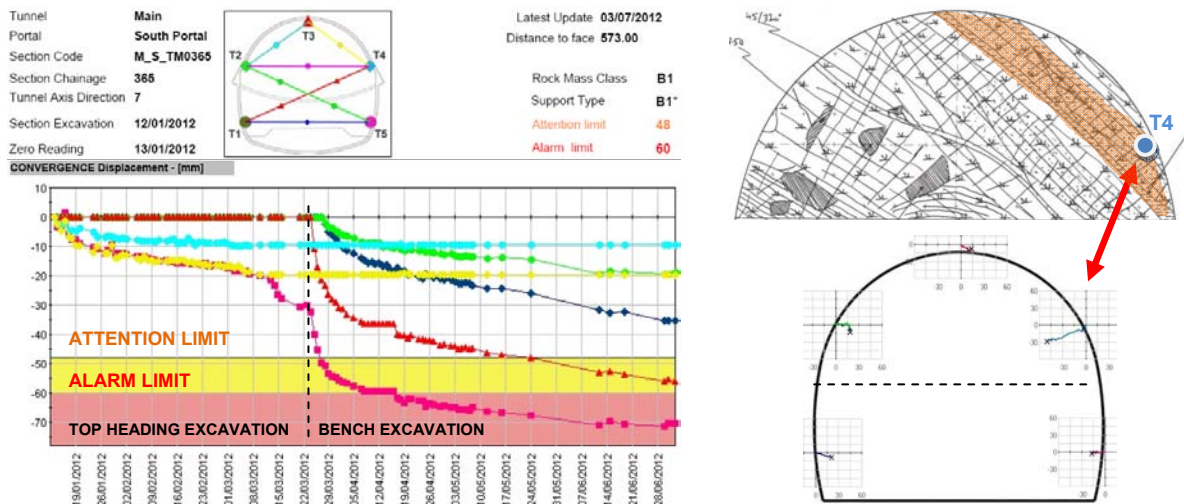
Figure 9. Geological face mapping correlated with the monitoring results at TM365

By exceeding both attention and alarm limits, along with the recorded cracks on the shotcrete, the monitoring data highlighted the necessity to strengthen the primary support and apply an heavier support class (B2 with 250mm FR shotcrete, lattice girders and rock bolts). This proved to be a very effective countermeasure, ensuring the control of the maximum displacement and bringing it down within the limits.

With reference to monitoring section at TM527 (Figure 10), where B2 support class was applied since excavation, deformations during the benching phase increased by "only" 2 times compared to those recorded during top heading excavation. In this case, the better response of the rock mass was reasonably due to the applied stronger support as well as to the prevalence of intermingled sandstone and siltstone against thin layers of claystone.
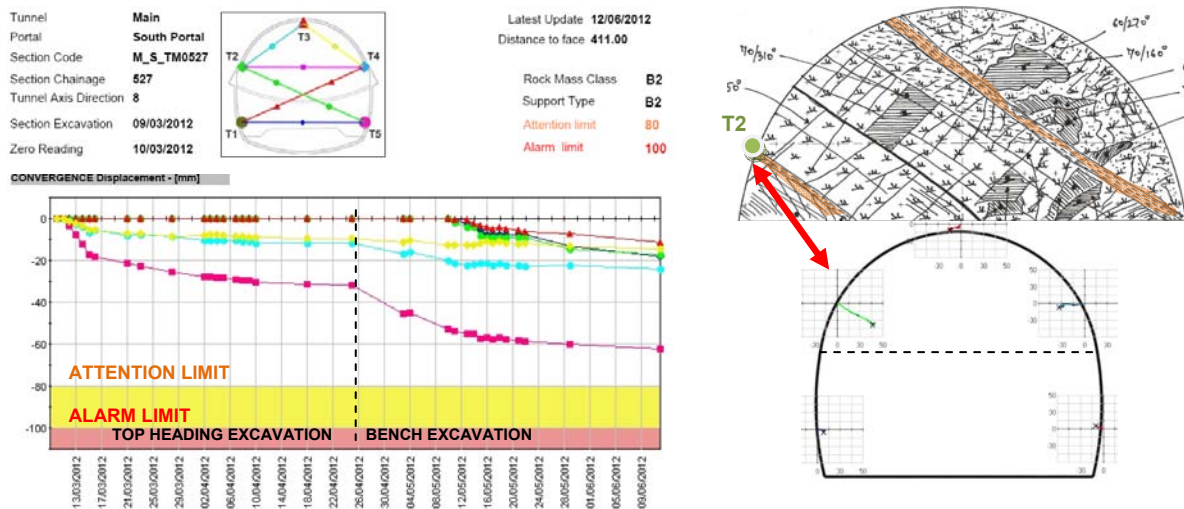

Figure 10. Geological face mapping correlated with the monitoring results at TM527

The usefulness of an accurate monitoring system is more evident when looking at the trend of the convergences measured along the main tunnel for a length of about 1.3 km from the south portal (Figure 11).

Constant and timely assessments of the monitoring data allow to select the proper support system to be installed and possibly define the required countermeasures by limiting the deformations which may affect the overall stability of the tunnel and lead to overload of the inner lining (which means additional reinforcement, cost and time consuming).
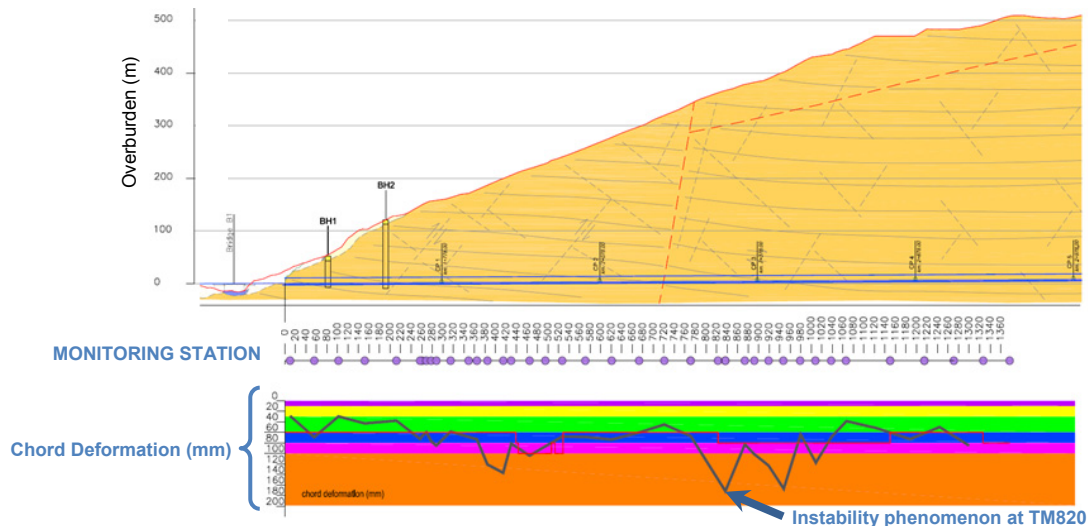
Figure 11. Trend of the convergences along the examined tunnel sector (countermeasures applied where convergence limits have been exceeded)

5   Conclusion

The behaviour of flysch formations cannot be easily determined due to the uncertainties in characterizing their anisotropy and heterogeneity. As well demonstrated by the results of the numerical analyses, the stability of the excavation in the flysch rock masses mainly depends on the prevalent expected lithology (sandstone, siltstone or claystone), on the geostructural conditions (favourable or unfavourable strata orientation) as well as the combined effect with the real state of stress. In this complicated medium, a proper monitoring system plays a fundamental role in highlighting potential risks and in selecting the most appropriate support system, which can thus help to optimize the overall excavation process and avoid time and cost consuming remedial interventions.

6   Acknowledgements

7   References

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